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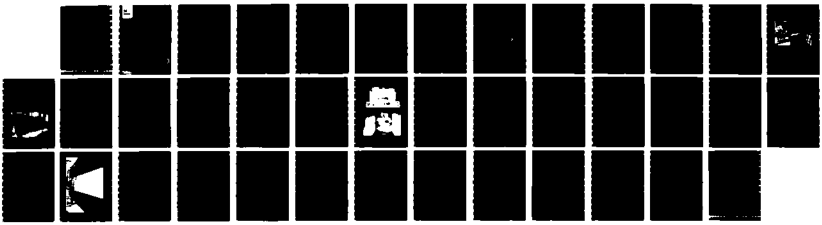

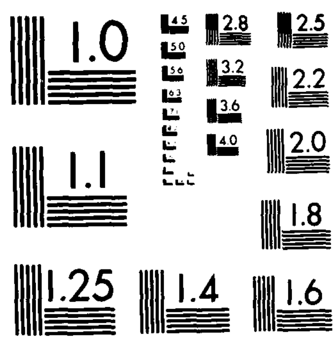

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TECHNICAL REPORT HL 86-8

McGEE CREEK PUMPING STATION SUMP PIKE COUNTY, ILLINOIS

Hydraulic Model Investigation

by

Glenn R. Triplett

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

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Final Report

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19 ABSTRACT (Continue on reverse if necessary and identify by block number)

The McGee Creek Pumping Station sump model study was conducted to evaluate the characteristics of inflow conditions and to develop modifications, if needed, to improve flow distribution to the pump intakes.

The operation of the 1:10.4-scale model of the original design sump showed uniform flow distribution from the trapezoidal channel to the pump bays. Reasonably good flow distribution existed in the bay approach to the individual pumps. Eddies were generated as the flow came through the constricted sluice gate openings. Diverging sidewalls streamlined the flow back into the bay area, but there were no converging sidewalls for streamlining the flow into the constricted sluice gate opening due to the position and design of the sluice gate. Some dissipation of the eddies occurred in the bay approach area, while circular motion continued to the pump column area where surface vortices occurred under certain operating conditions.

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19. ABSTRACT (Continued).

The intersump catwalk openings caused a problem when the water surface elevation was raised above el 421. Circular flow was generated as water flowed freely through these openings to adjacent sumps. This circular flow added to the problem from the eddies and gave strength to the formation of surface vortices.

Test results indicated no significant increase in adverse flow due to offcenter location of both side pumps in the original design. The original design intersump drain openings allowed some intersump flow, but its effect also was insignificant. This report does not advocate offcenter pump locations or intersump drain opening near the pump bell intake without a model study to determine their effect for a specific sump. These two irregular features (offcenter location of the pumps and intersump openings near the pump bell intake), combined with the eddy from the sluice gate openings, produced an overall adverse effect that was less than the adverse effects of some of the irregular features tested alone. The recommended design satisfactorily corrected the net adverse effects of these features.

Numerous modifications were tested to eliminate the circular flow and vortices. Doors for the intersump catwalk openings prevented the circular flow and vortices created by the open intersump catwalks at high sump water levels. Surface vortex suppressor beams eliminated all other surface vortices and provided a more even flow distribution to the pump intake.

Testing with the 30 percent increase in discharge for a range of submergences provided an array of data values for predicting future results for comparative operating parameters.

PREFACE

The model investigation of the McGee Creek Pumping Station sump reported herein was authorized by the US Army Engineer District, St. Louis (LMS), on 13 January 1981.

This investigation was conducted during the period January 1981 to November 1981 in the Hydraulics Laboratory of the US Army Engineer Waterways Experiment Station (WES), under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Mr. J. L. Grace, Jr., Chief of the Hydraulics Structures Division, and under the general supervision of Mr. N. R. Oswalt, Chief of the Spillways and Channels Branch. The project engineer for the model study was Mr. G. R. Triplett, assisted by Ms. L. Yates and Messrs. E. L. Jefferson and R. Bryant, Jr., all of the Spillways and Channels Branch. Mr. B. F. Stanfield, Engineering and Construction Services Division, is acknowledged for his work in constructing the model. This report was prepared by Mr. Triplett.

During the course of the study, Messrs. James Luther, Walter Wagner, and Ben Venturella of LMS; Emil Cook and Mark Wagner from the consulting firm of Crawford, Murphy, and Tilly, Inc.; Joe McCormick, Larry Eckenrod, Larry Cook, and Roddis C. Randall of the US Army Engineer Division, Lower Mississippi Valley; and John J. Robertson of the Office, Chief of Engineers, visited WES to discuss the program of model tests, observe the model in operation, and correlate test results with concurrent design work.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

CONTENTS

CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurements used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
acres	0.4047	hectares
cubic feet	0.02832	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
feet of water (39.2°F)	2,988.98	pascals
inches	2.54	centimetres
miles (US statute)	1.609	kilometres
pounds (force) per square inch	6.894757	kilopascals

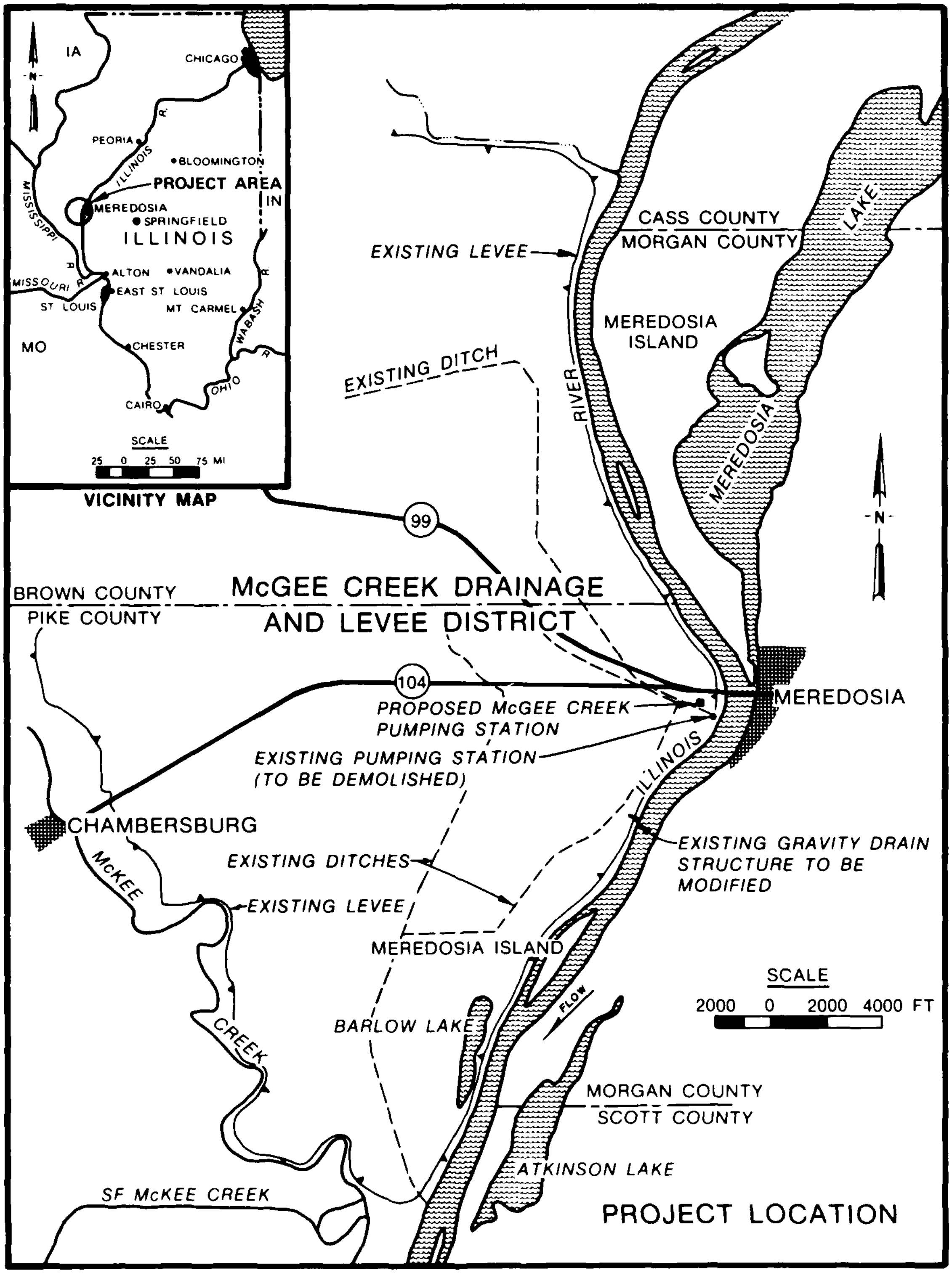

Figure 1. Location map

McGEE CREEK PUMPING STATION SUMP

PIKE COUNTY, ILLINOIS

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. The location for the proposed McGee Creek Pumping Station is in west-central Illinois, in the northeast corner of Pike County. The site is located on the west bank of the Illinois River about 1 mile* west of Meredosia, Illinois, and about 52 miles west of Springfield, Illinois (Figure 1).

2. The pumping station is part of a flood-control project that will provide protection to approximately 12,234 acres of predominantly agricultural lands in the McGee Creek Drainage and Levee District, which is located in Brown and Pike Counties, Illinois. The District includes the Illinois River bottomlands on the right bank between river miles 67.2 and 75.1 above the mouth of the Illinois River, bounded on the north by Kamp Creek and on the south and west by McGee Creek. The project provides for construction or reconstruction of 14.7 miles of levee, pumping station, and closure structure at Illinois State Highway 104 crossing.

3. An existing pumping station and gravity flow structure currently provide limited flood protection to the McGee Creek Drainage and Levee District. These structures cannot maintain a sufficiently low ground-water elevation during the growing season due to seepage through the Illinois River levee and rainfall runoff. Overbank flood damages begin at an elevation of 422.7,** and high water table damage begins at el 420.7. The existing pumping station has a capacity of 196 cfs and has maintained a mean annual flood elevation of 424.2 over a 40-year period of record. The gravity flow drain is essentially useless, except during winter months when water levels in the

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

** All elevations cited herein are in feet referred to the National Geodetic Vertical Datum (NGVD).

ditches are allowed to rise above those maintained during the growing season. The existing station will be demolished after completion of the new pumping station.

4. The proposed pumping station will be of the wet pit (sump) type employing three vertical propeller-type mixed or axial flow pumps operating with siphonic recovery. The design discharge for each pump at high operating heads is 107 cfs. At low operating head the expected discharge for each pump is 133 cfs. Proposed start-up sequence plans for the pumps provide for pump 1 to be started when the water level in the sump reaches el 419.5; pump 2 at el 420.5; and pump 3 at el 421.5. The plan provides for an automatic shutdown of the pumps at el 420.5, el 419.5, and el 418.0, respectively. The proposed station would maintain a mean annual flood elevation of 420.9. The mean annual pumping days would be reduced from 153 days to 67 days, and the mean annual full capacity days would be reduced from 95 days to 23 days.

5. The original design sump (Type I) consists of three individual compartments separated by straight divider walls that extend upstream more than 35 ft to the 9- by 9-ft sluice gate openings. The straight sidewalls also extend upstream an additional 9.5 ft from the sluice gate openings. The sump floor has approximately 6 in. of downward slope from the sluice gate to the backwall to facilitate sump draining. Intersump drain openings 2 ft wide by 1 ft high are located flush with the floor and 1 ft from the backwall. Intersump catwalk openings (3 ft 10 in. wide by 7 ft 6 in. high) are located just downstream from the pump as well as upstream from the pumps (4 ft wide by 7 ft 6 in. high). The approach flow enters the sump from more than 300 ft of straight trapezoidal channel transitioned by sheet pile wing walls 24 ft long at 45-deg angles. The center pump is located on the center line of the middle sump compartment. Each of the two side pumps is located 3 in. off center line of their individual sump compartments toward the outside wall of the sump. At the time of testing, pump selection by the St. Louis District had been reduced to two manufacturers. For testing purposes, the more restrictive of the two configurations was used (60-in. pump bell). The pumps were located 2.5 ft above the sump floor and 11 in. from the back wall (Figure 2). Plates 1 and 2 illustrate the general sump arrangement. Sluice gates are provided in each of the three compartments to allow the sumps to be dewatered and cleaned. Wooden stop logs are also provided for use when a sluice gate is out of service due to a failure or for maintenance. All flows are screened by a trashrack prior

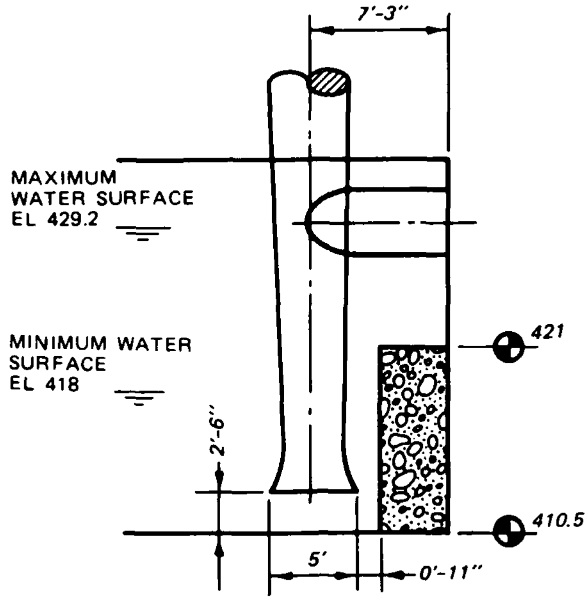


Figure 2. Pump location

to entering the pump intake area. Each pump will discharge over the Illinois River levee through a 48-in.-diam steel pipe with siphonic recovery. The flow profile is shown in Figure 3. A model study of the siphonic recovery system was conducted at the US Army Engineer Waterways Experiment Station (WES) concurrently with this study. The results of this related study may be found in WES (1982).*

Purpose of Model Study

6. The model study was conducted to evaluate the characteristics of inflow to the original sump and to develop modifications for improving the flow distribution to the pump intakes if needed.

* Ronald R. Copeland. 1982 (Sep). "McGee Creek Pumping Station Siphon, Hydraulic Model Study," Technical Report HL-82-23, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

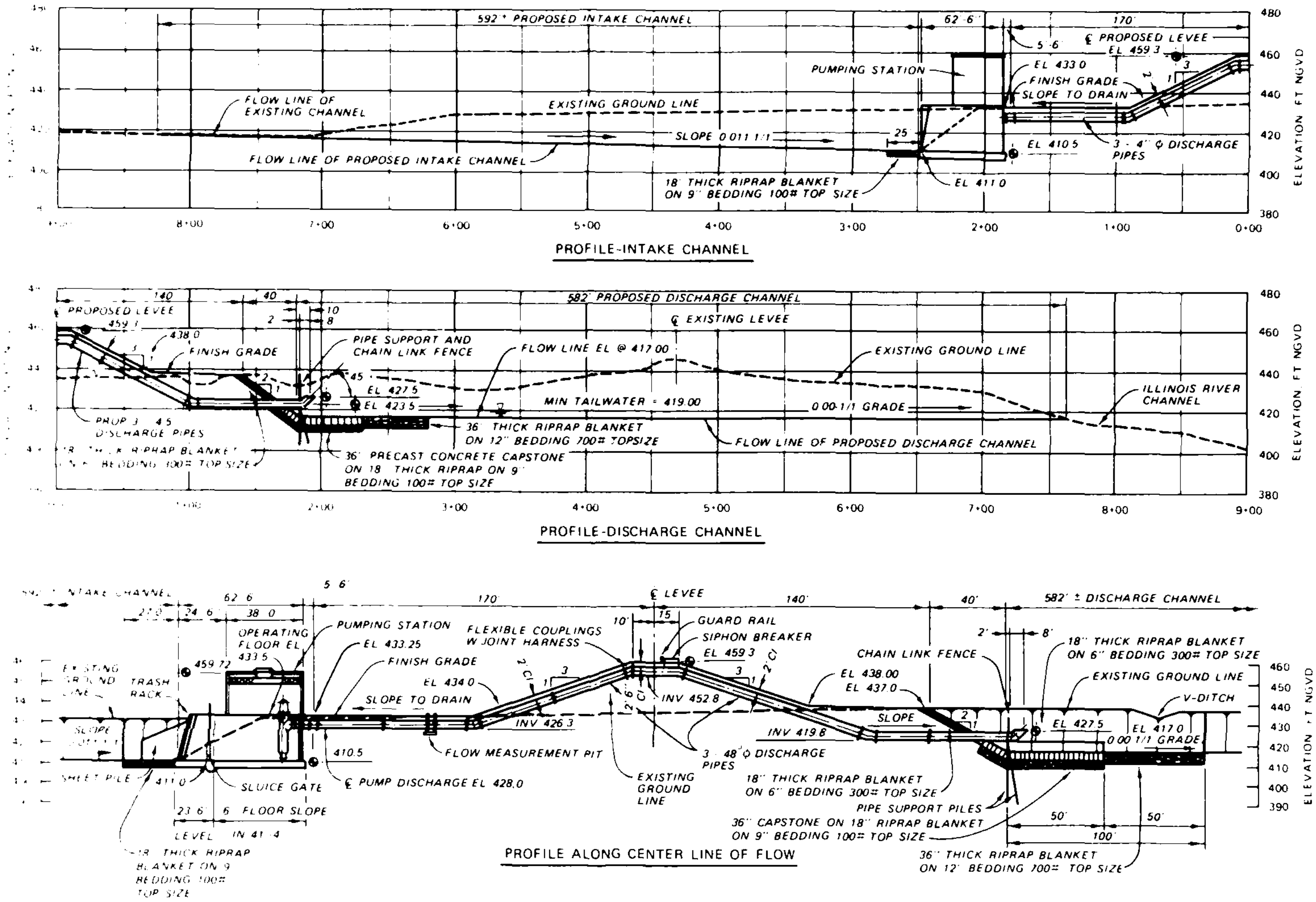


Figure 3. Flow profiles

PART II: THE MODEL

Description

7. The model was constructed to an undistorted linear scale ratio of 1:10.4 (Figure 4). The model reproduced an area over 300 ft long by 150 ft

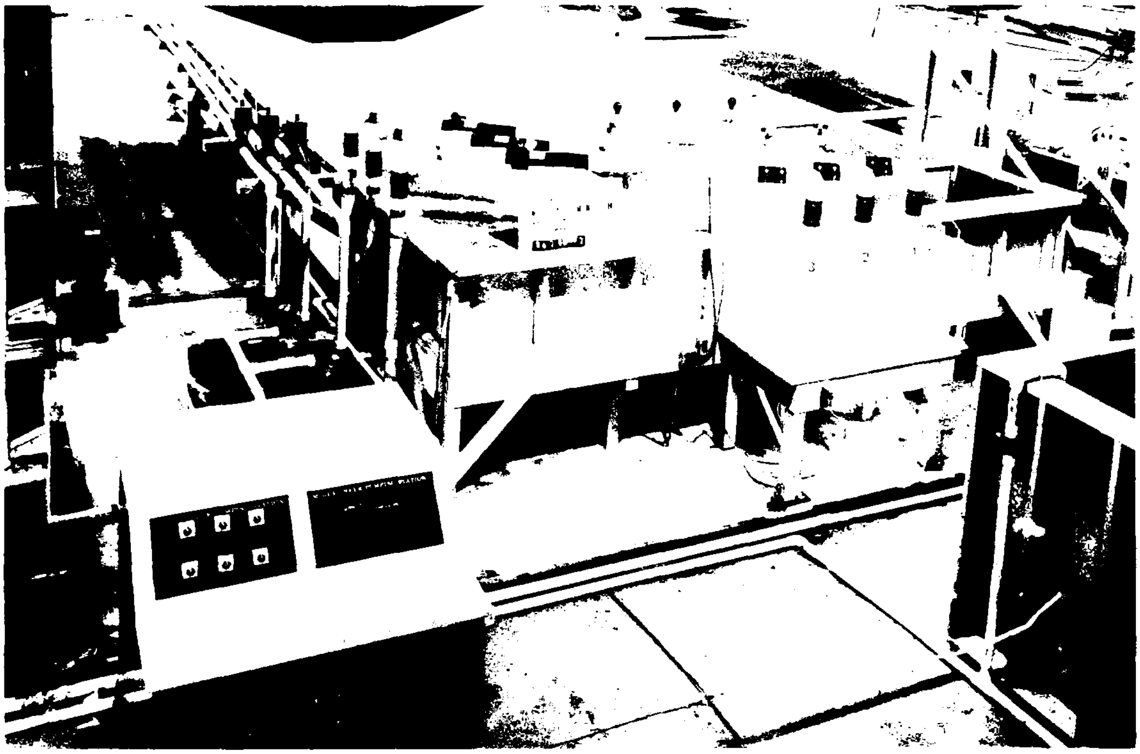


Figure 4. General view of model

wide of the channel, the entire sump, and the three pump intakes (Figure 5). The sump and pump intakes were constructed of transparent plastic to permit observation of flow currents, turbulence, vortices, and rotational flow characteristics. Transparent measurement scales were attached to the side of the sump to show both model water level and prototype elevation. Flow through each of the suction bell intakes was provided by a centrifugal suction pump located on the floor beside the model. The discharge through each suction bell was measured by "paddle wheel" flowmeters and regulated by valves, all installed in the piping downstream from the suction bells. The water was recirculated to the upstream end of the flume where it was baffled (Figure 6) before reentering the channel. Alternate piping with appropriate valves was

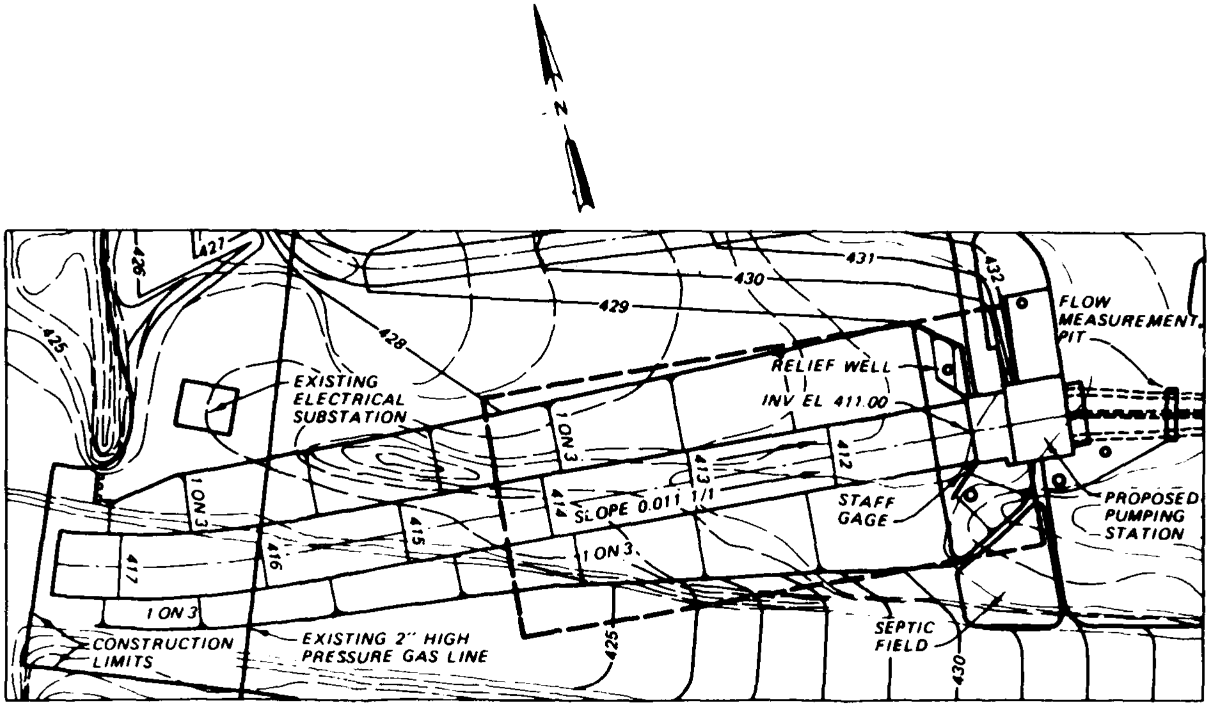


Figure 5. Limits of model

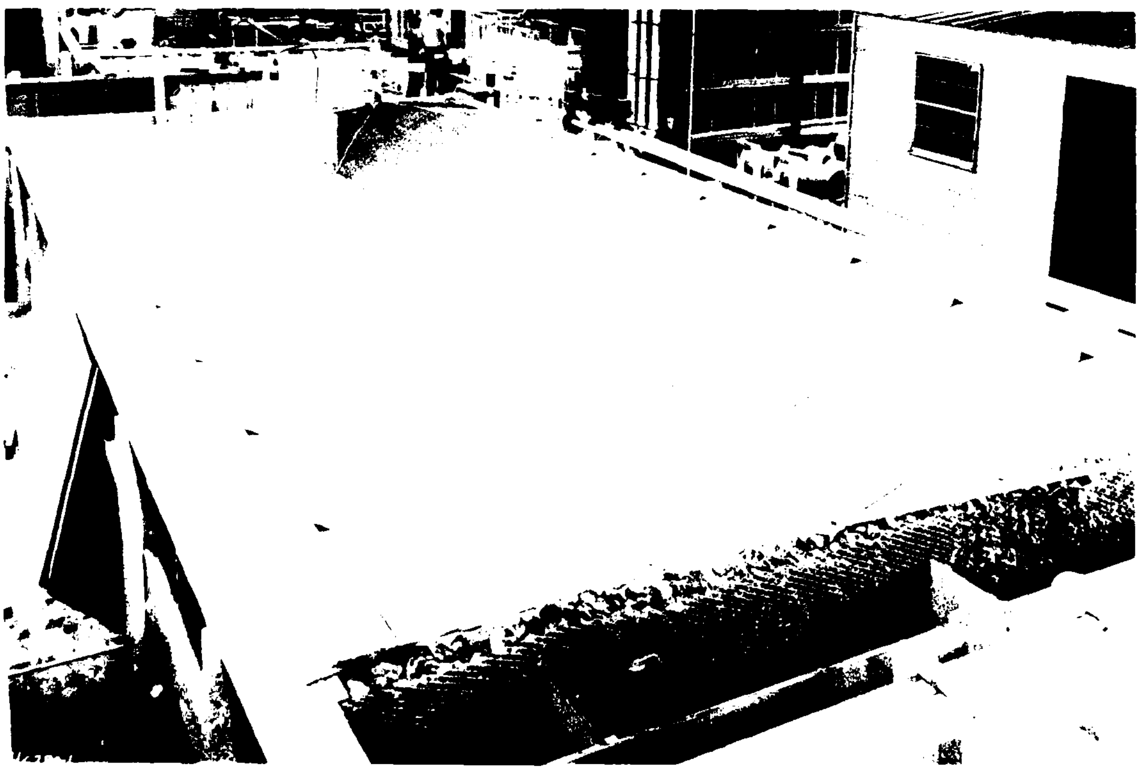


Figure 6. General view of model showing baffled inflow area

available to pump the water to a reservoir where volumetric verification of discharge accuracy was made.

8. An instrument control console was conveniently located beside the sump so that the model operator could monitor the gages while also recording observations of flow conditions. Special lighting was located beneath each pump intake and above the overall sump to improve visibility of flow conditions. A light installed above each pump column was lit to indicate which pumps were in operation. Flow patterns were determined by observation of dye injected into the water at various elevations and by confetti sprinkled on the water surface. Adverse flow conditions were also measured in the model by vortimeters and pressure transducers. The vortimeter, a free-wheeling propeller with four zero-pitched blades (Figure 7), was used to measure the rotational flow (swirl) and was located in approximately the same position as the impeller in the axial/mixed-flow pump (prototype). The 5-psi pressure transducer with 5.2-in.-diam (prototype) diaphragm was face mounted flush with the sump floor under the center point of each suction bell (Figure 7). Velocities were measured with an electromagnetic velocity probe. The trashracks were simulated by a grid constructed of thin plastic strips. Adjustable plastic sheets were installed at the sluice gate openings to allow testing at partial gate openings to determine if partially open sluice gates could serve a dual purpose to suppress surface vortices as well as isolate sumps. Adjustable sliding plastic doors were also provided to close the intersump drain openings and intersump catwalk openings.

Interpretation of Model Results

9. The size of the physical model was sufficiently large such that Reynolds numbers of flows were approximately 10^5 or greater to minimize scale effects due to viscous forces, where

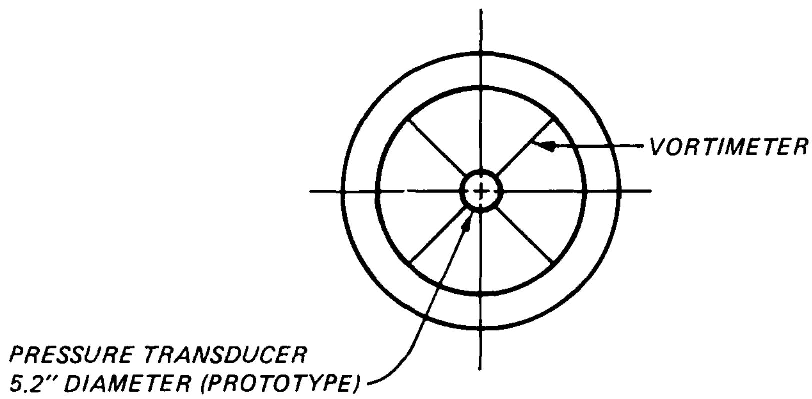
$$R = \frac{Vd}{\gamma} \tag{1}$$

where

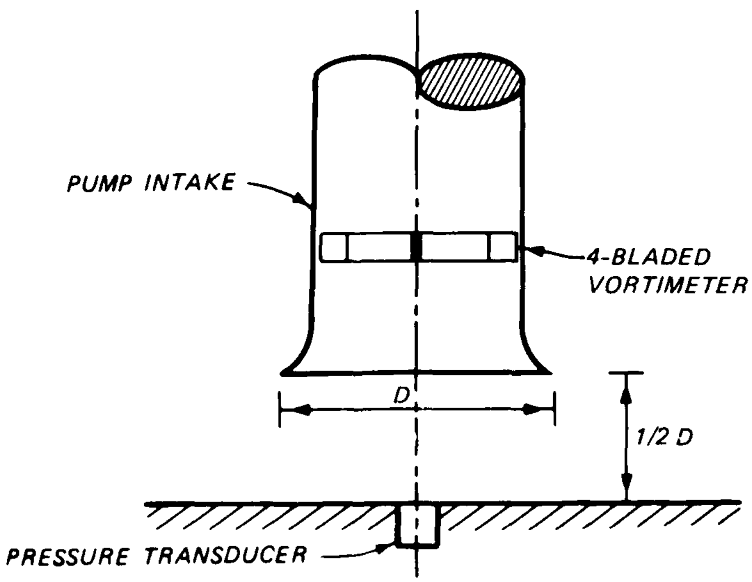
V = average velocity in pump column

d = diameter of pump suction pipe

γ = kinematic viscosity of water



PLAN VIEW



ELEVATION

Figure 7. Pressure transducer and vortimeter locations

Accepted equations of hydraulic similitude based on the Froudian criteria were used to express the mathematical relationships between the dimensions and hydraulic quantities of the model and prototype in terms of the model scale or length ratio, as follows:

Dimension	Model/Prototype Ratio	Model to Prototype Scale Relationship
Length	$L_r = L$	1:10.4
Area	$A_r = L^2$	1:108.16
Volume	$V_r = L^3$	1:1,124.86
Pressure	$P_r = L$	1:10.4
Discharge	$Q = L^{5/2}$	1:348.81
Time	$T_r = L^{1/2}$	1:3.22
Frequency	$f_r = \frac{1}{L^{1/2}}$	1:0.31

Values for discharge, water surface elevation, dimensions, frequency, etc., can be converted quantitatively from the model value to the prototype equivalent by use of these scale factors.

PART III: TESTS AND RESULTS

Original Design

10. The original design (Type I) sump was tested for all possible combinations in which the three pumps could operate. The expected sequence of pump operation was stated in paragraph 4. The pump combinations were operated for the full range of expected water surface elevations, 418 ft minimum to 428.2 ft maximum; however, data were recorded only at el 418, 422, and 428.2 ft since these levels included worst-case flow conditions (Table 1). Surface vortices occurred in the original design at all three submergences. The more advanced vortices (Stage E) occurred at the lowest water level (el 418). Figure 8 defines the stages of surface vortex development that are listed in Table 1. Figures 9 and 10 show some typical surface vortices. Figure 9 shows the vortex without dye, and Figure 10 shows dye inserted into the vortex core. The maximum pressure fluctuation P_f beneath the suction bells was 5 ft. The maximum value for the rotational flow indicator R_i was 0.19. The rotational flow indicator is a dimensionless parameter derived from the vortimeter rotations. The indicator was calculated by the equation

$$R_i = \frac{U}{V_a} \tag{2}$$

where U is the blade speed and is determined from the equation

$$U = \frac{\pi b n}{60} \tag{3}$$

where

b = vortimeter blade diameter

n = vortimeter revolutions per minute

The average axial velocity V_a is calculated from the following equation:

$$V_a = \frac{Q}{A} \tag{4}$$

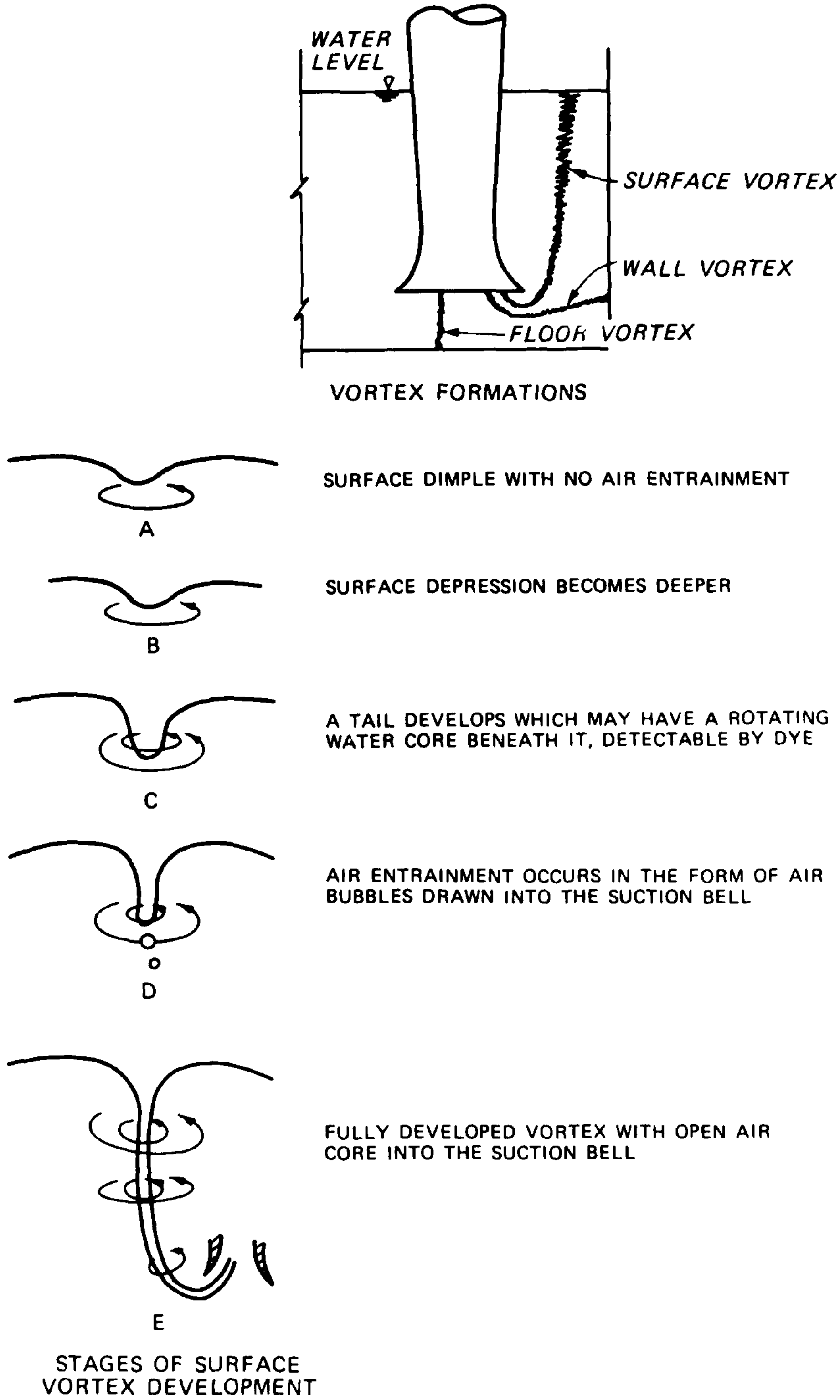


Figure 8. Definitions of vortex formations and stages of surface vortex development

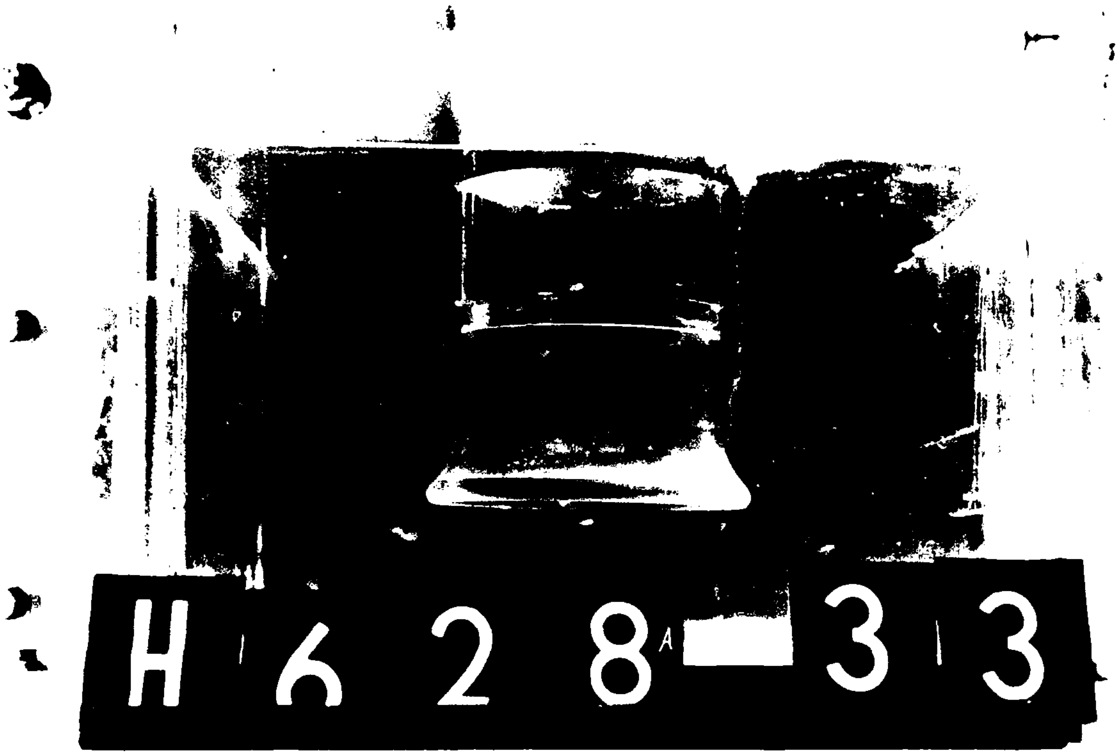


Figure 9. Typical surface vortex in the original design sump

Figure 10. Typical vortex (with dye insertion) in the original design sump

where

Q = discharge, cfs

A = cross-sectional area of the pump column (inside diameter) at the blade location.

11. The flow pitch F_p (Table 1) is a measure of the amount of spiral in the flow at the approximate location of the pump impeller and is a calculated value of the number of pump bell diameters the flow will travel axially while completing one spiral revolution. F_p is based on the number of vortimeter revolutions, the cross-sectional area of the pump column (inside diameter), and the pump bell diameter. It is determined as follows:

$$F_p = 2\pi D \frac{V_a}{U} \tag{5}$$

where D is the pump bell diameter and V_a and U are as defined in paragraph 10. The most adverse F_p (153) occurred in test run 3 of the original design tests.

12. In Table 1, the data are recorded according to which combination of pumps is operating. The numbering sequence is the same as that in the prototype: 1, 2, 3 from left to right looking downstream.

13. Approach velocities were measured 6 ft upstream from the center line of the pump column and 1 ft above the sump floor. All pumps were operating at 134 cfs. Plate 3 shows the measured velocities for water surface elevations of 418 and 422. The velocity values indicate a reasonably good flow distribution to the pump bell intake in the original design sump.

Experimental Designs

14. The Type II design sump was the same as the original without the 45-deg sheet piling wing walls. Wing walls were incorporated during an earlier design review meeting; however, test results determined that they were unnecessary. Wing walls have proven to be very effective in streamlining flow in projects where the channel flow approaches from an angle to the sump and when sump divider walls are very short. Velocities were sufficiently slow in the approach flow that the test results showed no significant difference between the Type II and the Type I (original design) sumps.

15. Guide vanes are similar to surface vortex suppressor beams (SVSBs) which were later added to the recommended design. The guide vanes (Type III design sump) were sloped downward to divert flow toward the bell intake. The guide vanes were effective, but the location required for optimum performance was not satisfactory.

16. SVSBs (Type IV sump design) were effective, and a satisfactory location was determined. The beams alone, however, were not completely satisfactory at the higher water levels where the catwalk openings allowed a swirl condition to develop.

17. Partially open sluice gates (sump design Type V) were tested to determine if they would produce results as satisfactory as those of the SVSBs. The partially open gates did produce favorable results, but the degree of the opening had to be changed when the surface water elevation was changed. No single fixed gate opening was found that would provide satisfactory results at all expected water surface elevations.

18. Intersump drain openings and intersump catwalk openings were tested in the open and closed positions (Type VI and VII designs, respectively). The change in the drain opening produced no significant difference in flow conditions; however, the flow conditions were substantially improved when the intersump catwalk openings were closed for the high water surface elevations.

19. Each of the two outside pumps were located 3 in. offcenter in the original design (Plate 1). The pumps were relocated to a centered position (Type VIII design) for testing. The difference in flow conditions between the pump offcenter positions and the oncenter positions was insignificant.

20. Converging sidewalls (Type IX design) were tested but provided little improvement in flow conditions. The SVSBs provided considerably more improvement in flow conditions than the converging sidewalls and should be less costly to install. Other research studies have shown that converging sidewalls are more beneficial when adverse approach flow conditions prevail.

Recommended Design

21. SVSBs of various heights were tested at numerous positions both vertically (above the floor) and horizontally (with respect to the distance upstream from the pump). All these various sizes and locations of SVSBs were tested for all possible combinations of pumps operating and water surface

elevations between 416 ft and 428.2 ft. The recommended design (Type X) sump (Plate 4) required three SVSBs (one per each sump) and four catwalk doors (two each per intersump wall) to provide adequate flow distribution to the pump intakes. The SVSBs improved flow conditions at expected water surface elevations. The catwalk doors were needed only at water levels above el 421.0. Table 2 gives the test data for the recommended design corresponding to that for the original design, Table 1. In the recommended design, no surface or subsurface vortices occurred. Other comparisons of pressure fluctuations P_f beneath the pump bell and rotational flow are shown by bar charts in Figures 11 and 12, respectively. In the recommended design the maximum P_f was 1.9 ft, which occurred at the high water surface el (428.2 ft) when all three pumps were operating at a discharge of 134 cfs each. This fluctuation (1.9 ft) is compared with the maximum P_f of 5 ft in the original design sump. A much larger percentage differential is noted in the maximum values of the rotational flow indicator R_i, which was 0.040 in the recommended design sump as compared with 0.205 in the original design. The minimum number of diameters of axial flow per spiral flow revolution F_p was increased from 153 in the original design sump to 783 in the recommended design sump.

22. Approach velocities were measured in the recommended design sump at the same location as in the original design sump (6 ft upstream from the center line of the pump column and 1 ft above the sump floor). The results are shown in Plate 5 for water surface el 418 and 422. The approach velocities were again reasonably uniform indicating good approach flow distribution to the pump bell intakes. The velocities were larger in some areas of the recommended design sump compared with the original design sump velocities. This difference is apparently due to the downward deflection and obstruction effect of the SVSBs.

Increased Flow Rates

23. Additional tests were conducted at a discharge of 176 cfs with the recommended design. This discharge is an increase of 30 percent in the flow rate from previous tests. These tests were also conducted for all combinations of pumps operating, as in previous testing. Three additional water surface elevations were tested at the increased flow rate, making a total of six levels (428.2, 422, 420, 418, 417, and 416) tested for all combinations of

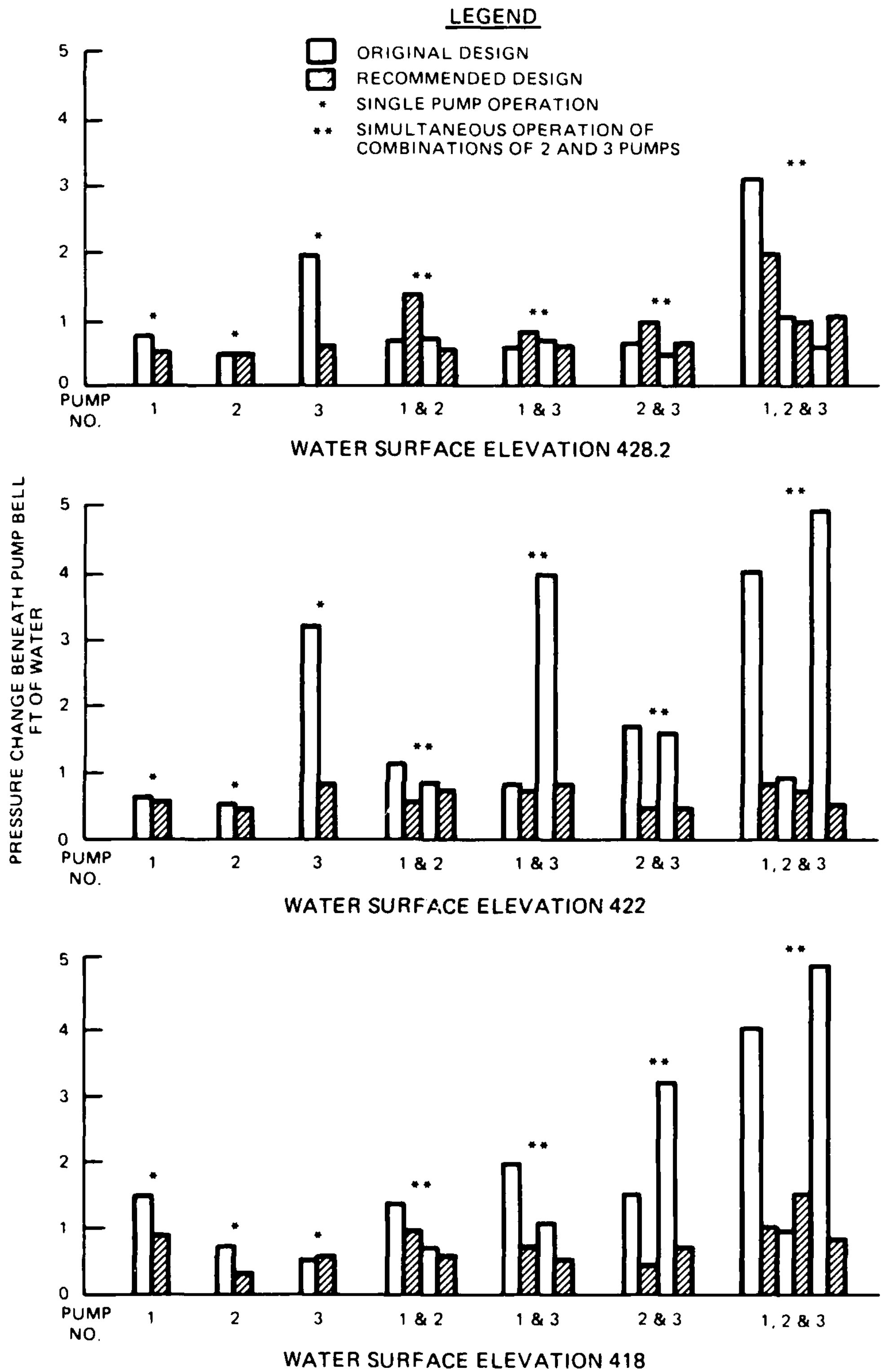


Figure 11. Pressure changes beneath the pump bell during pump discharge of 134 cfs for all pump operating combinations and for three water surface elevations

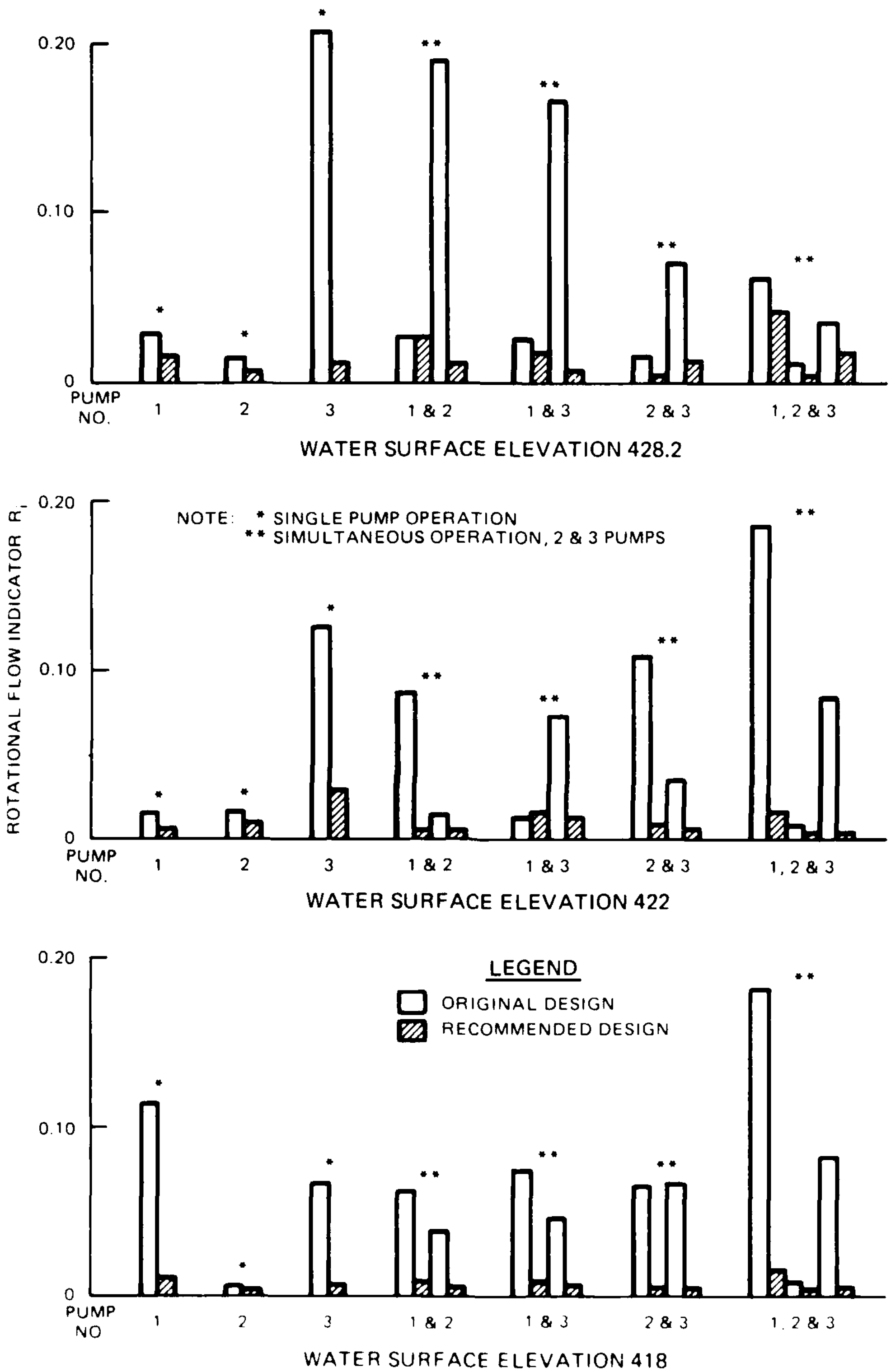


Figure 12. Rotational flow versus combinations of pumps operating. Discharge 134 cfs at maximum water surface el 428.2, 422, and 418

pumps operating (Table 3). In addition, two lower water surface elevations (415 and 414) were tested for pump 3 only.

24. Experience has shown trends for adverse flow development with an increase in pump intake velocity or a decrease in pump bell submergence. The data in Table 3 show this trend beginning with test runs 22-28 at the 30 percent increased discharge for the expected maximum water surface elevation (428.2). The maximum pressure fluctuation beneath the pump bell increased to 2.7 ft compared with 1.9 ft at the lower discharge (134 cfs). The rotational flow indicator increased to 0.046 compared with 0.040 for the lower discharge. Small changes in pump bell intake velocity or submergence may not cause large changes in flow conditions if the discharge and submergence are already conservatively ranged; however, this series of tests demonstrated the pertinent range of submergence at which adverse flow conditions develop more rapidly.

25. At water surface el 418 and discharge 176 cfs, surface vortices (stages A and B) began to occur, the maximum rotational flow indicator R_i was 0.061, and the maximum pressure fluctuation P_f was 2.2 ft.

26. At water surface el 417, stage E surface vortices developed, the maximum R_i was 0.158, and the maximum P_f was 2.7 ft.

27. At water surface el 416, stage E vortices occurred almost continually, the maximum R_i was 0.235, and the maximum P_f was 5 ft.

28. Adverse flow conditions due to the low water levels were so dramatic at water surface el 415 and 414 that only one pump was operated with the increased flow rate (176 cfs). At el 415, stage E vortices occurred, R_i was 0.103, and P_f was 1.5 ft. At el 414 stage E surface vortices occurred and also intermittent subsurface vortices occurred between the suction bell and the floor. The P_f was 1.5 ft, but data could not be obtained to calculate R_i due to the large amount of air being drawn into the pump column preventing accurate vortimeter readings.

29. The increased flow rate test results may be more clearly visualized from Figure 13 where R_i and P_f are plotted against the pump bell submergence. Both values begin to increase rapidly as the water surface elevation is decreased below 418 ft. Pump bell submergence is shown in both feet and pump bell diameters to provide a more visual comparison of this application with those of other authors.

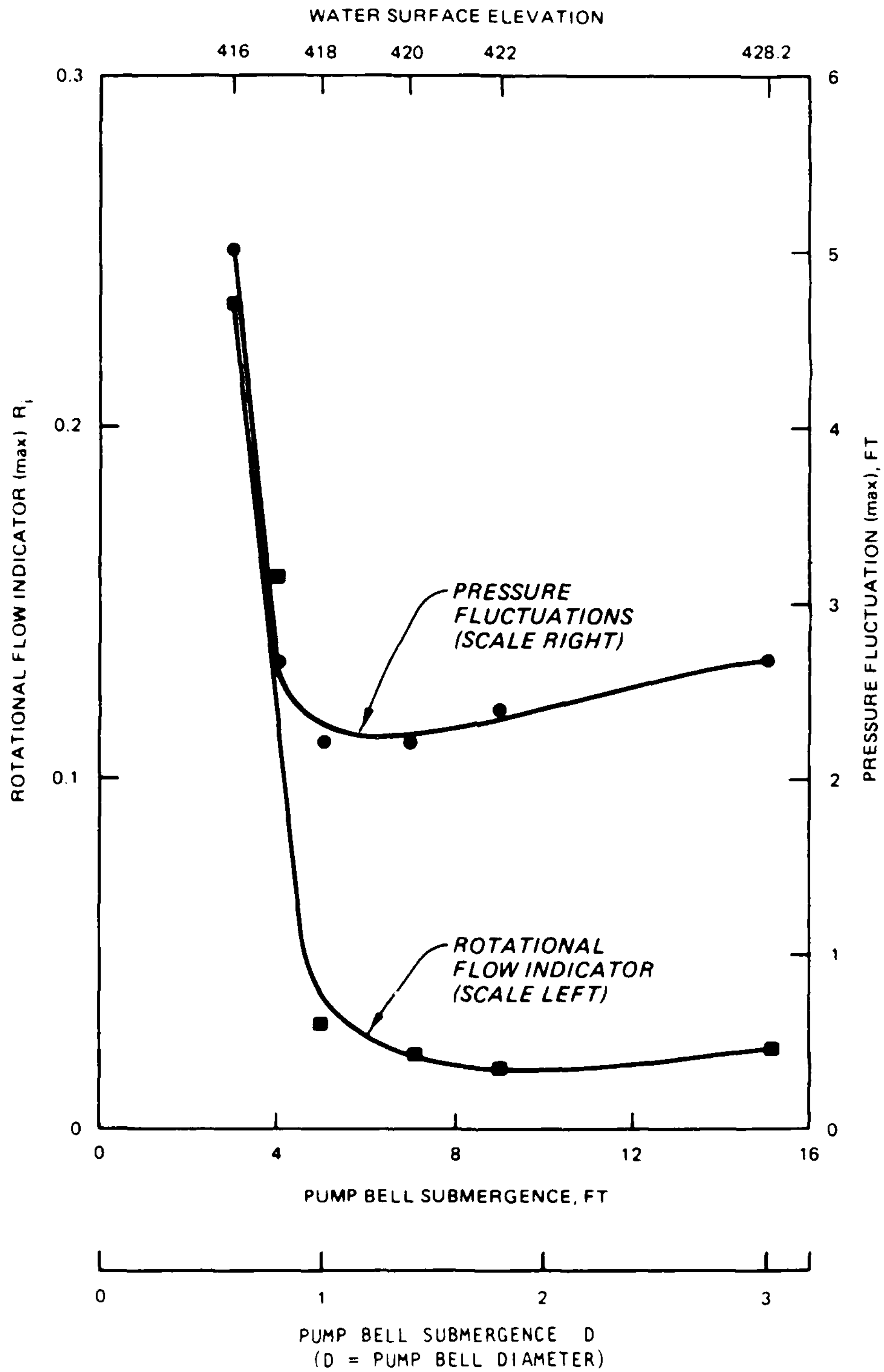


Figure 13. Pressure fluctuations and rotational flow indicator versus submergence for the increased flow rate, 176 cfs

PART IV: CONCLUSIONS AND RECOMMENDATIONS

30. The original McGee Creek Pumping Station sump and approach flow design provided the general characteristics necessary for good inflow to the pump bell intakes. The long, straight, trapezoidal approach channel provided uniform flow distribution to the pump bays (Figure 14). The rectangular sump bays with long divider walls (approximately seven pump bell diameters) provided additional streamlining of flow to the individual pumps. It is generally recognized that six times the pump bell diameter (6 D) is a good design length for the sump. The location of pump intakes is nearly ideal according to WES research,* which indicates the pump bell intake should be located approximately 1/4 D from the backwall, 1/2 D from the floor, and 1/2 to 1 D from the sidewalls. The submergence for McGee Creek varies from 1 to 3 D which remains within an acceptable operating range as indicated by Figure 13, which shows the pressure fluctuations and rotational flow indicators versus submergence for the 30 percent increased flow rates.

31. The 9-ft-sq sluice gate openings caused some disruptive constriction in flow, particularly at the higher water levels. The original design provided diverging sidewalls (see Plate 1) to allow the flow transition back to the larger bay width. On the upstream side of the gate, no converging slope was provided, apparently due to the position and design of the sluice gate. This 90-deg angle in the wall surface adjacent to the entrance of the sluice gate allowed eddies to develop at the entrance of the sluice gate opening. The eddies were partially dissipated within a few feet of the gate opening. Most of the remaining circular motion was removed by the SVSBs. Numerous locations of SVSBs were tested to determine the optimum streamlining of flow and elimination of surface vortices.

32. The offcenter location of the two side pumps (3 in. toward the outside walls) and the intersump drain openings (2 ft by 1 ft) caused very minor changes in the uniform flow obtained with the SVSBs. No changes were recommended for either the original pump location or the intersump drain openings.

33. The 3-ft-wide intersump catwalk openings, located two places in each inside wall, cause a considerable amount of circular motion when the

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* Glenn R. Triplett et al. "Pumping Station Inflow-Discharge Hydraulics, Generalized Pump Sump Research Study" (In preparation), US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

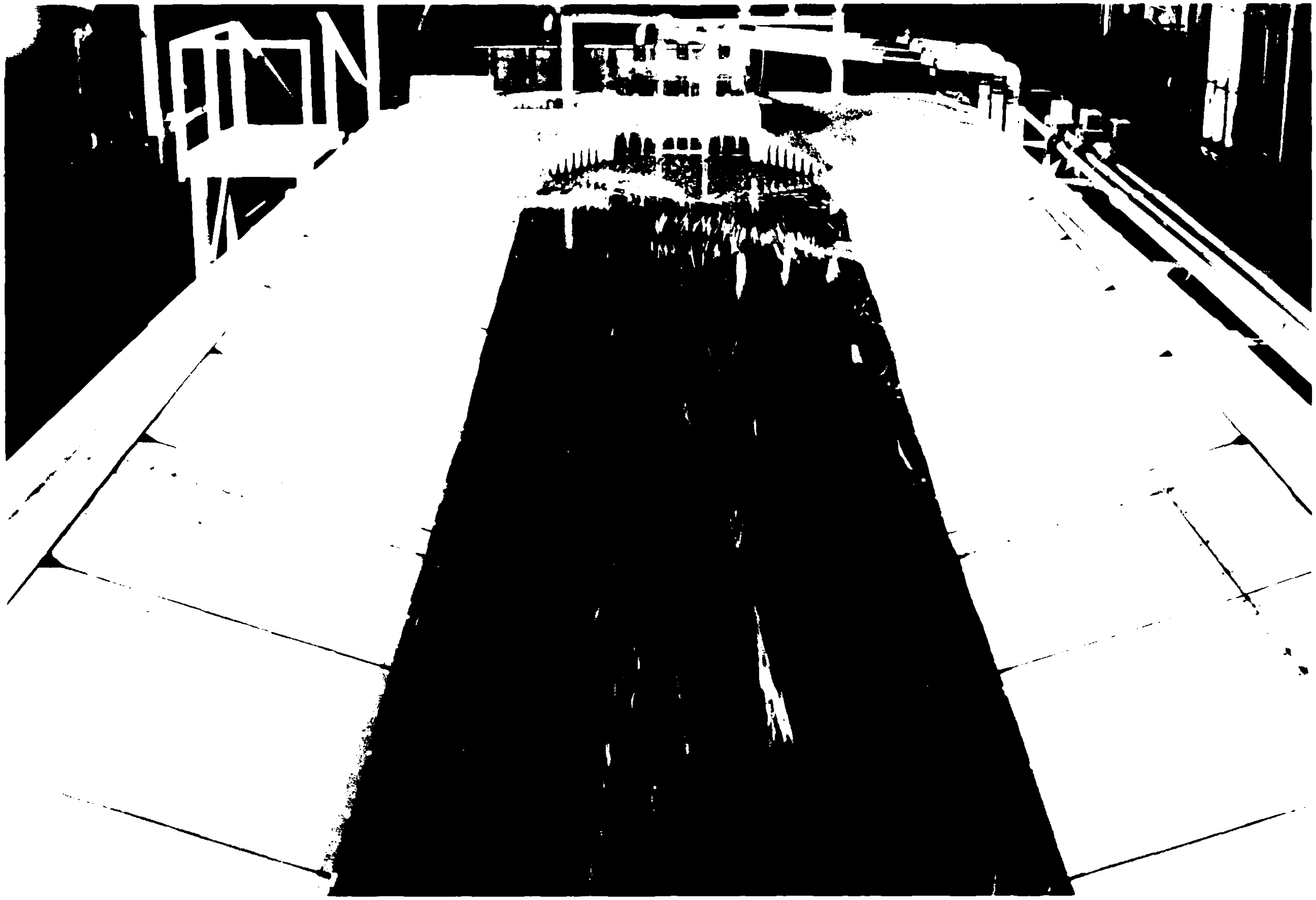


Figure 14. Approach flow pattern of recommended design with all pumps operating

water level rises above el 421. It was recommended that doors be installed in these openings so that they can be closed when water levels above el 421 are expected.

34. The pumping station was completed prior to final publication of this report, permitting observation of the station in operation. During this particular 1-day field evaluation, good performance was observed (free of cavitation noise, vibrations, surface vortices, and unsymmetrical flow conditions) under normal pumping rates of 133 cfs. Various combinations of one, two, and three pumps operating at sump levels between el 418.5 and 420.5 were observed. High sump levels were not obtainable on this day of observation; however, satisfactory performance was obtained on other occasions at higher sump levels except when excessive debris was trapped in the sump.

Table 1

Pressure Fluctuations, Flow Pitch, Rotational Flow Indicator, and Vortices

Original Design

Test Run No.	Discharge per Pump Q, cfs	Water Surface El	Flow Pitch F_p D/Rev Pump 1	Pump 2	Pump 3	Rotational Flow Indicator R_i, Dimensionless Pump 1	Pump 2	Pump 3	Surface Vortices, Stage Pump 1	Pump 2	Pump 3	Subsurface Vortices	Pressure Fluctuations ft of water Pump 1	Pump 2	Pump 3
1	134	428.2	1,208+	x	x	0.026+	x	x	None	None	None	None	0.7	x	x
2			x	2,618-	x	x	0.012-	x	None	A	None		x	0.4	x
3			x	x	153+	x	x	0.205+	None	None	None		x	x	1.9
4			1,208+	165-	x	0.026+	0.190-	x	None	None	None		0.7	0.7	x
5			1,208-	x	188+	0.026-	x	0.167+	None	None	None		0.6	x	0.7
6			x	2,244+	449+	x	0.014+	0.070+	None	None	None		x	0.7	0.4
7			515+	3,491+	898+	0.061+	0.009+	0.035+	None	None	None		3.2	1.0	0.6
8		422.0	2,618+	x	x	0.012+	x	x	None	None	None		0.7	x	x
9			x	2,244+	x	x	0.014+	x	None	D	None		x	0.6	x
10			x	x	249+	x	x	0.126+	None	None	None		x	x	3.2
11			361-	2,244+	x	0.087-	0.014+	x	None	None	None		1.0	0.8	x
12			2,618+	x	430+	0.012+	x	0.073+	A	None	B		0.8	x	3.9
13			x	288+	898+	x	0.109+	0.035+	None	None	A		x	1.7	1.6
14			170+	3,491+	383-	0.185+	0.009+	0.082-	None	None	None		4.0	0.9	4.9
15		418.0	281+	x	x	0.112+	x	x	E	None	None		1.4	x	x
16			x	15,710+	x	x	0.002+	x	None	None	None		x	0.7	x
17			x	x	483-	x	x	0.065-	None	None	None		x	x	0.5
18			515+	827+	x	0.061+	0.038-	x	B	None	None		1.3	0.7	x
19			430+	x	669+	0.073+	x	0.047+	A	None	C		1.9	x	1.1
20			x	483-	462-	x	0.065-	0.068-	None	None	E		x	1.5	3.2
21			173+	3,491-	383-	0.182+	0.009-	0.082-	E	B	D		4.0	0.9	4.9

Note: x = a pump not operating (shown in pertinent data column); + = clockwise rotation; - = counterclockwise rotation. Stages of surface vortex development are specified by letters A through E with E being worst condition with a fully developed air core. See Figure 8 for a more complete description.

Table 2

Pressure Fluctuations, Flow Pitch, Rotational Flow Indicator, and Vortices

Recommended (Type X) Design

Test Run No.	Discharge per Pump Q, cfs	Water Surface El	Flow Pitch F_p D/Rev Pump 1	Flow Pitch Pump 2	Flow Pitch Pump 3	Rotational Flow Indicator R_i, Dimensionless Pump 1	Rotational Flow Pump 2	Rotational Flow Pump 3	Surface Vortices, Stage Pump 1	Surface Vortices Pump 2	Surface Vortices Pump 3	Subsurface Vortices	Pressure Fluctuations ft of water Pump 1	Pressure Fluctuations Pump 2	Pressure Fluctuations Pump 3
1	134	428.2	2,191+	x	x	0.014+	x	x	None	None	None	None	0.4	x	x
2			x	10,957-	x	x	0.003-	x					x	0.4	x
3			x	x	2,739+	x	x	0.011+					x	x	0.6
4			1,217+	3,130-	x	0.026+	0.010-	x					1.3	0.5	x
5			1,826+	x	5,479+	0.017+	x	0.006+					0.8	x	0.6
6			x	10,957+	2,435+	x	0.003+	0.013+					x	0.9	0.7
7			783+	10,957+	1,826-	0.040+	0.003+	0.017-					1.9	0.9	1.0
8		422.0	7,305+	x	x	0.004+	x	x					0.6	x	x
9			x	3,652-	x	x	0.009+	x					x	0.4	x
10			x	x	1,096+	x	x	0.029+					x	x	0.8
11			5,479+	5,479+	x	0.006+	0.006+	x					0.6	0.7	x
12			2,191+	x	2,739+	0.014+	x	0.011+					0.7	x	0.8
13			x	3,652+	5,479+	x	0.009+	0.006+					x	0.4	0.4
14			1,826+	7,305+	7,305-	0.017+	0.004+	0.004-					0.8	0.7	0.5
15		418.0	3,131+	x	x	0.010+	x	x					0.8	x	x
16			x	21,914+	x	x	0.001+	x					x	0.3	x
17			x	x	10,957+	x	x	0.004+					x	x	0.6
18			3,652+	10,957-	x	0.009+	0.004-	x					0.9	0.6	x
19			3,652+	x	3,652-	0.009+	x	0.009-					0.7	x	0.5
20			x	7,304-	7,304-	x	0.004-	0.004-					x	0.4	0.7
21			1,826+	21,914-	5,479+	0.017+	0.001-	0.006+					1.0	1.5	0.8

Note: x = a pump not operating (shown in pertinent data column); + = clockwise rotation; - = counterclockwise rotation. Stages of surface vortex development are specified by letters A through E with E being worst condition with a fully developed air core. See Figure 8 for a more complete description.

Table 3

Pressure Fluctuations, Flow Pitch, Rotational Flow Indicator, and Vortices

Recommended (Type X) Design Tested with 130 Percent Design Discharge

Test Run No.	Discharge per Pump Q, cfs	Water Surface El	Flow Pitch F_p D/Rev Pump 1	Pump 2	Pump 3	Rotational Flow Indicator R_1, Dimensionless Pump 1	Pump 2	Pump 3	Surface Vortices, Stage Pump 1	Pump 2	Pump 3	Subsurface Vortices	Pressure Fluctuations ft of water Pump 1	Pump 2	Pump 3
22	176	428.2	730-	x	x	0.043-	x	x	None	None	None	None	2.5	x	x
23			x	10,957+	x	x	0.003+	x					x	0.5	x
24			x	x	843+	x	x	0.037+					x	x	0.9
25			843+	1,370-	x	0.037+	0.023-	x					2.2	0.5	x
26			685+	x	1,217-	0.046+	x	0.026-					0.5	x	0.7
27			x	843+	1,565+	x	0.037+	0.020+					x	2.7	1.1
28			391+	996+	10,957+	0.080+	0.032+	0.003+					1.1	1.1	1.1
29		422.0	1,217+	x	x	0.026+	x	x					0.9	x	x
30			x	3,652+	x	x	0.009+	x					x	2.4	x
31			x	x	3,652+	x	x	0.009+					x	x	0.9
32			3,652+	1,370-	x	0.009+	0.023-	x					1.1	0.8	x
33			1,565+	x	1,565-	0.020+	x	0.020-					1.1	x	0.8
34			x	1,217+	3,652-	x	0.026+	0.009-					x	0.6	0.9
35			1,096+	996+	1,826-	0.029+	0.032+	0.017-					0.8	0.9	0.5
36		420.0	3,131-	x	x	0.010-	x	x					0.4	x	x
37			x	5,479-	x	x	0.006-	x					x	0.8	x
38			x	x	1,217-	x	x	0.026-					x	x	2.2
39			1,370-	1,217+	x	0.023-	0.026+	x					0.6	0.9	x
40			783+	x	7,304-	0.040+	x	0.004-					0.7	x	1.1
41			x	1,661+	1,289+	x	0.022+	0.024-					x	1.5	0.9
42			1,096+	21,915+	1,826-	0.029+	0.001+	0.017-					0.7	0.9	0.9

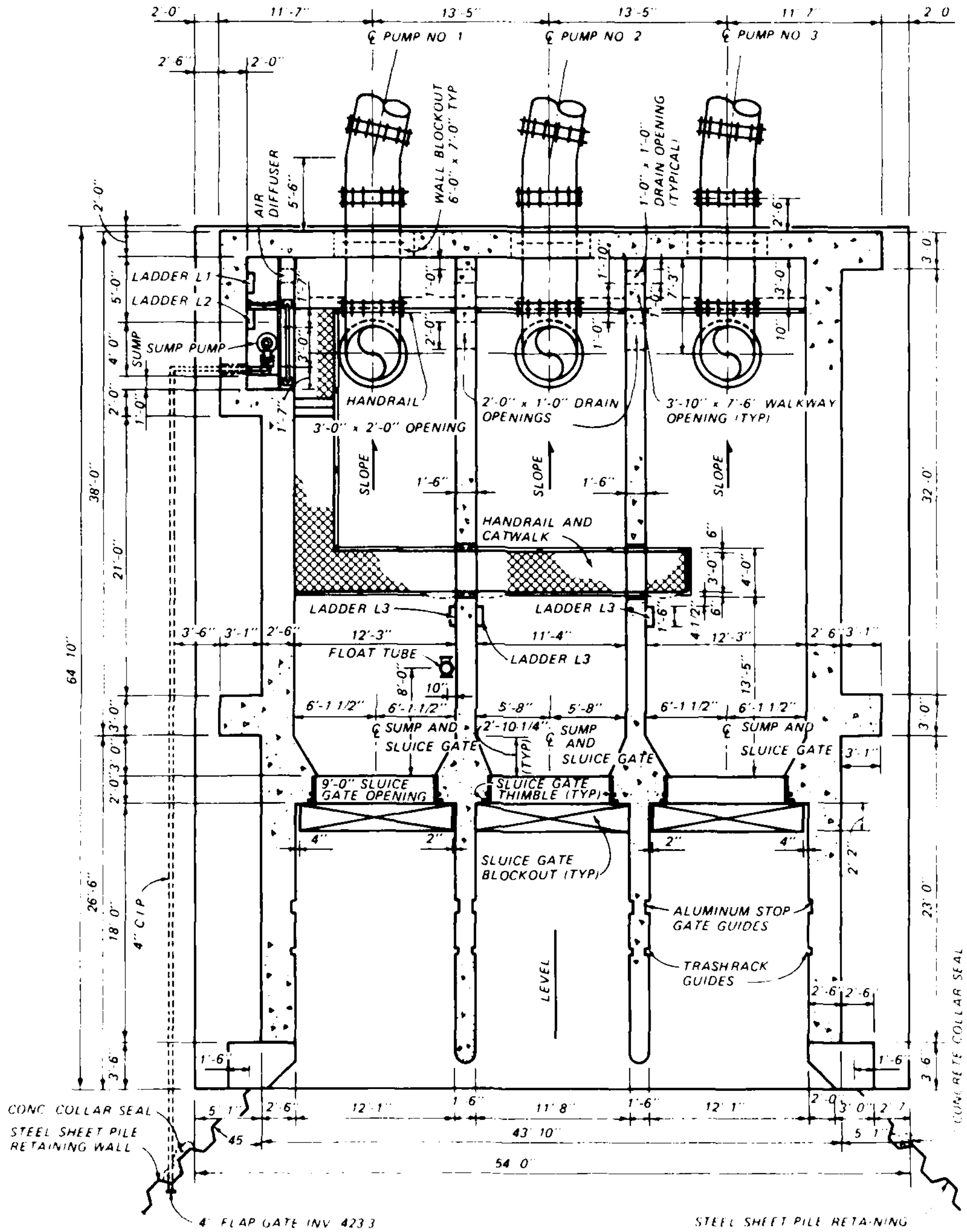
(Continued)

Note: x = a pump not operating (shown in pertinent data column); + = clockwise rotation; - = counterclockwise rotation. Stages of surface vortex development are specified by letters A through E with E being worst condition with a fully developed air core. See Figure 8 for a more complete description.

Table 3 (Concluded)

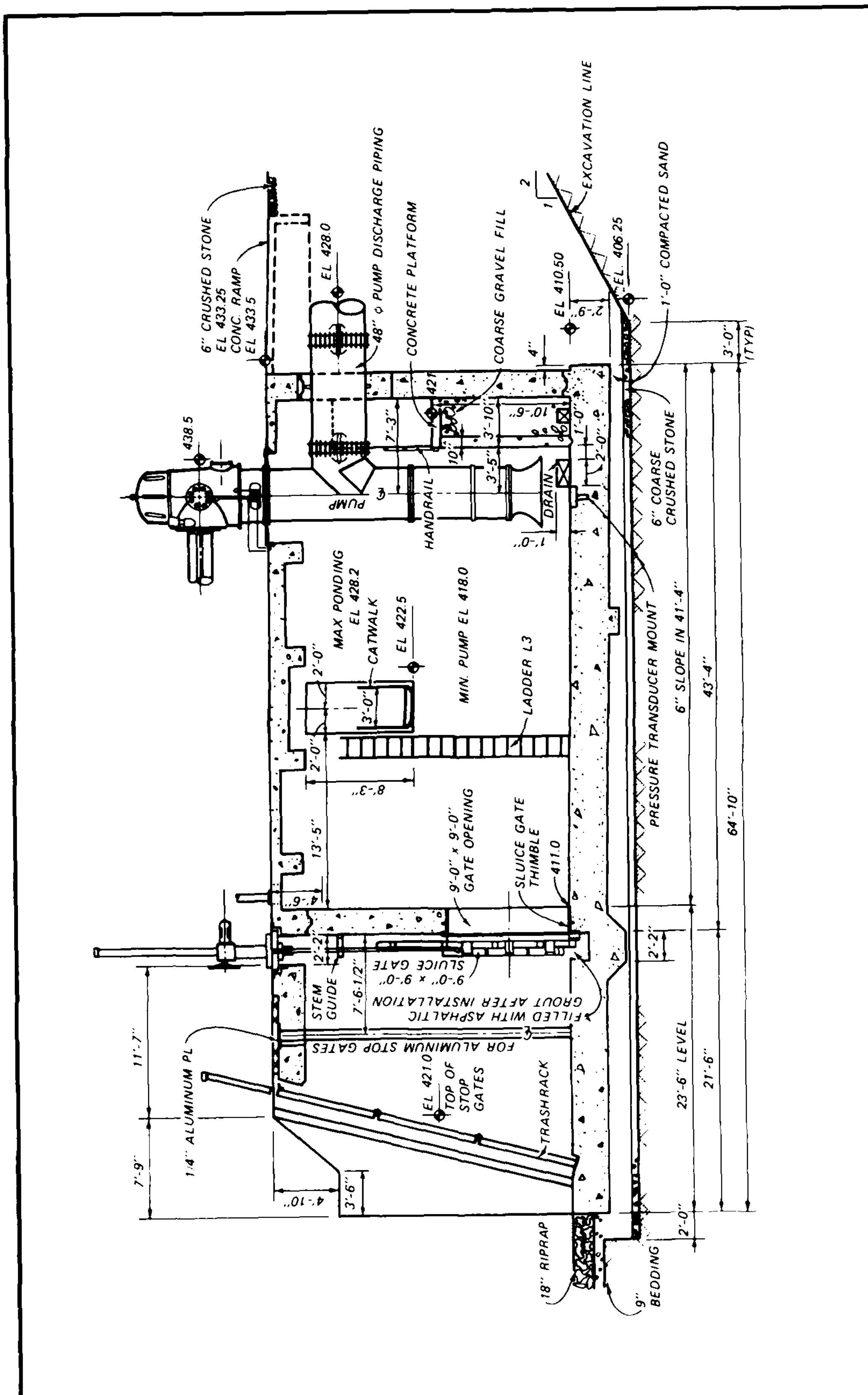
Test Run No.	Discharge per Pump Q, cfs	Water Surface El	Flow Pitch F_p D/Rev Pump 1	Pump 2	Pump 3	Rotational Flow Indicator R_i, Dimensionless Pump 1	Pump 2	Pump 3	Surface Vortices, Stage Pump 1	Pump 2	Pump 3	Subsurface Vortices	Pressure Fluctuations ft of water Pump 1	Pump 2	Pump 3
43	176	418.0	498-	x	x	0.061+	x	x	A	None	None	None	0.9	x	x
44			x	21,915+	x	x	0.001+	x	None	None	None		x	0.8	x
45			x	x	1,565-	x	x	0.020-	None	None	A		x	x	2.2
46			913+	21,915-	x	0.034+	0.001-	x	B	A	None		0.8	0.6	x
47			1,217+	x	2,739-	0.026+	x	0.012-	None	None	B		1.0	x	0.6
48			x	21,915-	843+	x	0.001-	0.037+	None	None	None		x	0.3	0.6
49			685+	5,479-	3,652-	0.046+	0.006-	0.009-	None	None	None		1.0	1.0	0.9
50		417.0	199-	x	x	0.158-	x	x	E	None	None		2.7	x	x
51			x	1,096-	x	x	0.029-	x	None	C	None		x	0.6	x
52			x	x	1,096+	x	x	0.029+	None	None	C		x	x	1.4
53			215+	1,370+	x	0.146+	0.023+	x	E	C	None		2.7	0.4	x
54			228+	x	684-	0.138+	x	0.046+	C	None	C		0.5	x	0.3
55			x	1,826-	365-	x	0.017-	0.086-	None	C	C		x	0.2	2.6
56			684-	645-	274-	0.046-	0.049-	0.115-	B	B	D		1.8	0.2	1.6
57		416.0	233-	x	x	0.135-	x	x	E	None	None		0.8	x	x
58			x	548-	x	x	0.057-	x	None	E	None		x	0.2	x
59			x	x	457-	x	x	0.069-	None	None	D		x	x	0.8
60			219-	547-	x	0.143-	0.057-	x	E	E	None		0.2	0.5	x
61			219-	x	685+	0.143-	x	0.056+	E	None	E		0.4	x	2.8
62			x	134-	365+	x	0.235-	0.086+	None	E	E		x	5.0	2.9
63			146+	274-	137+	0.215+	0.115-	0.229+	E	E	E		0.8	2.9	2.8
64		415.0	x	x	304-	x	x	0.103	None	None	E	None	x	x	1.5
65		414.0	x	x	*			*	None	None	E	To floor	x	x	1.5

* Ingested air was too large to allow freewheeling movement of the vortimeter.



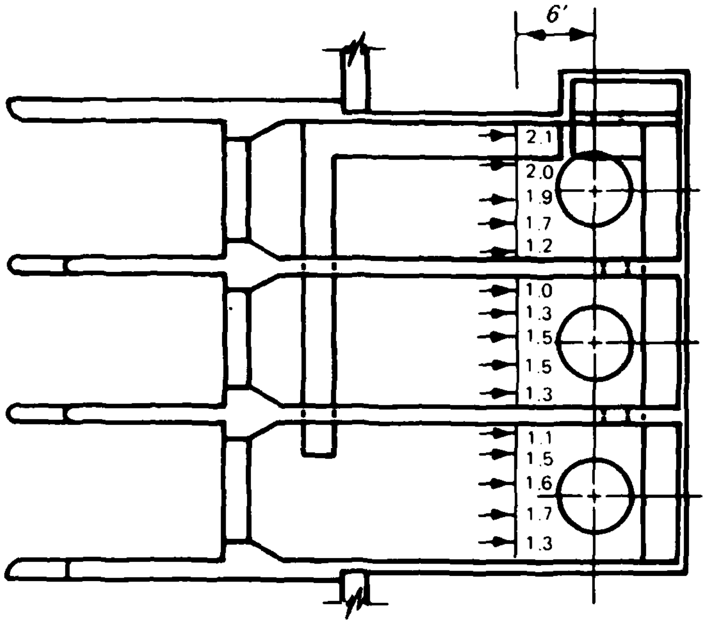
PUMP NO 1
PUMP NO 2
PUMP NO 3
AIR DIFFUSER
WALL BLOCKOUT 6'-0" x 7'-0" TYP
1'-0" x 1'-0" DRAIN OPENING (TYPICAL)
LADDER L1
LADDER L2
SUMP PUMP
SUMP
HANDRAIL
3'-0" x 2'-0" OPENING
2'-0" x 1'-0" DRAIN OPENINGS
3'-10" x 7'-6" WALKWAY OPENING (TYP)
SLOPE
HANDRAIL AND CATWALK
LADDER L3
FLOAT TUBE
SUMP AND SLUICE GATE
9'-0" SLUICE GATE OPENING
SLUICE GATE THIMBLE (TYP)
SLUICE GATE BLOCKOUT (TYP)
ALUMINUM STOP GATE GUIDES
TRASHRACK GUIDES
LEVEL
4" CIP
CONC COLLAR SEAL
STEEL SHEET PILE RETAINING WALL
4" FLAP GATE INV 423.3
STEEL SHEET PILE RETAINING WALL
CONCRETE COLLAR SEAL

PLAN VIEW

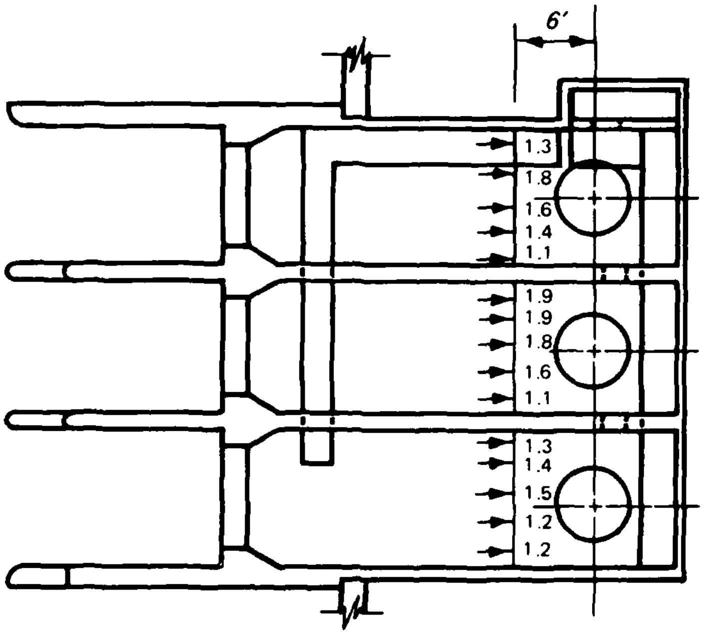


LONGTITUDINAL SECTION VIEW

PLATE 2



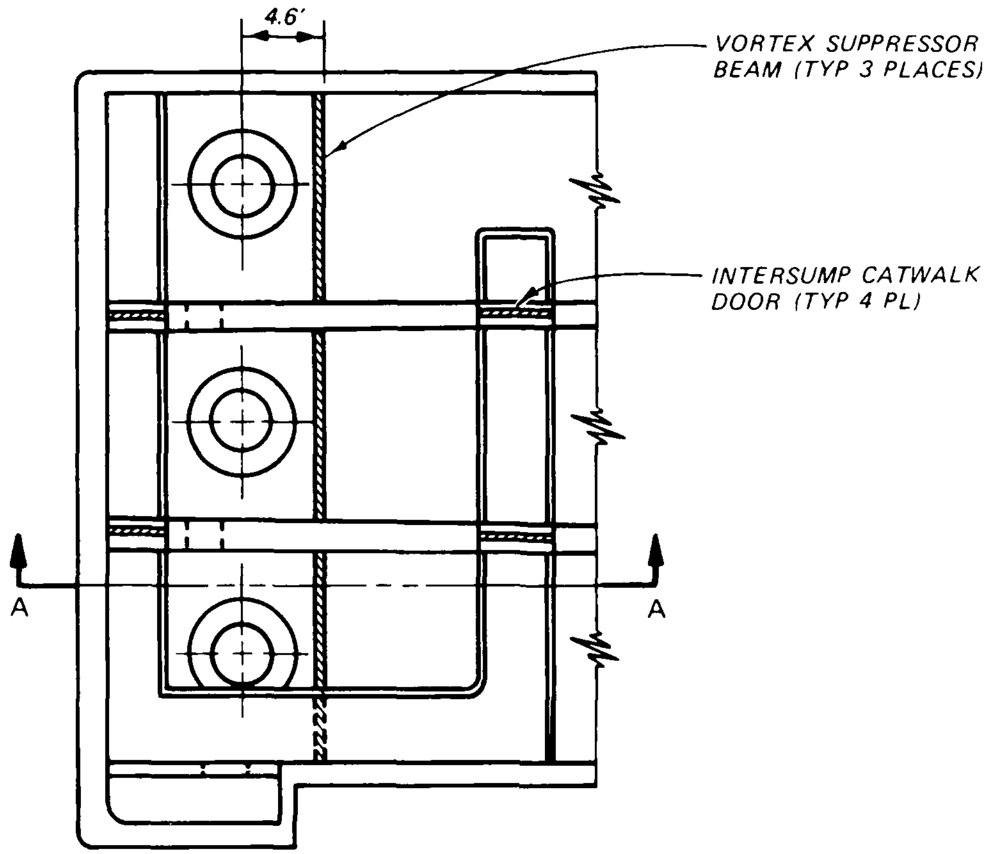
WATER SURFACE EL 418



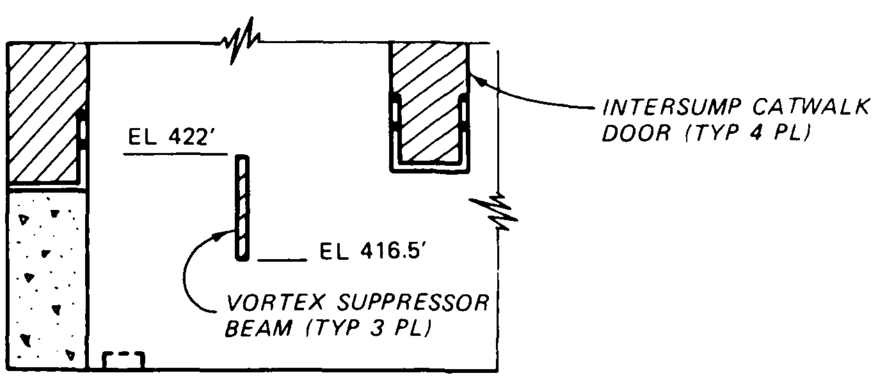
WATER SURFACE EL 422

NOTE: VELOCITIES GIVEN IN FEET PER SECOND.

APPROACH VELOCITIES
MEASURED 1 FT ABOVE SUMP FLOOR
ALL PUMPS OPERATING AT 134 CFS EACH
ORIGINAL DESIGN SUMP

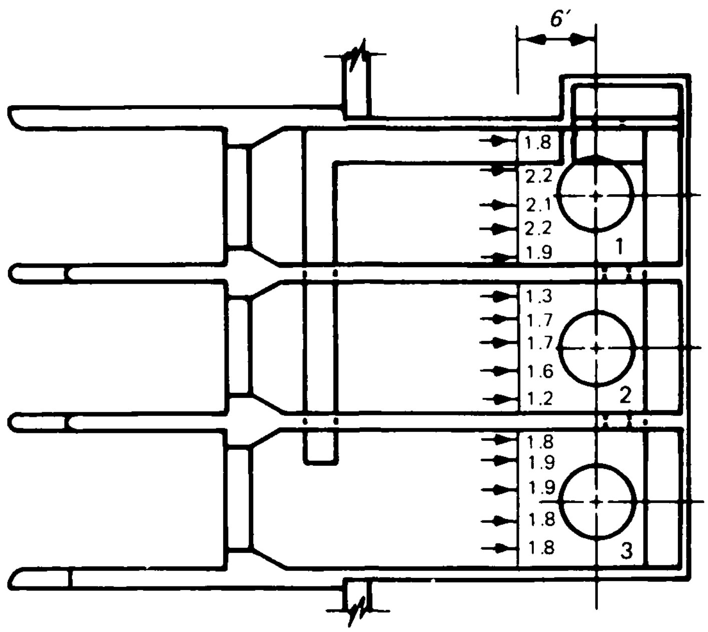


PLAN VIEW (PARTIAL)
(NOT TO SCALE)

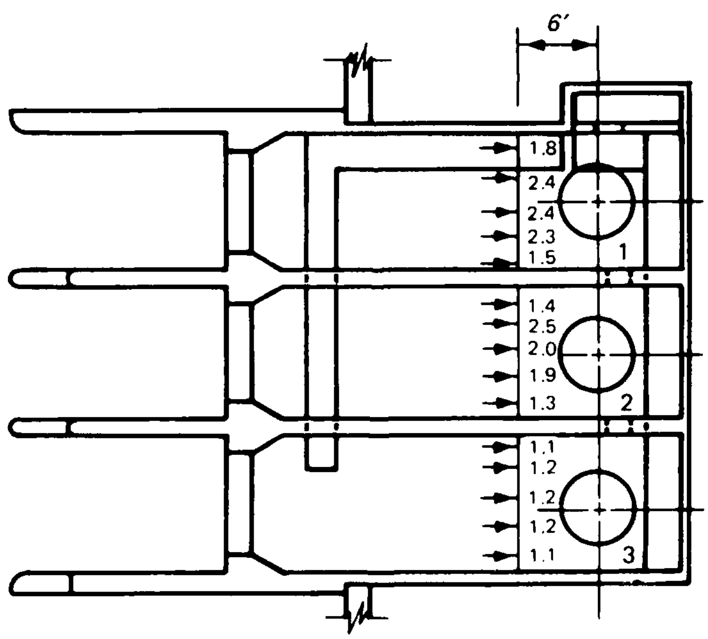


VIEW A-A (PARTIAL)
(NOT TO SCALE)

RECOMMENDED DESIGN WITH
VORTEX SUPPRESSOR BEAMS AND
INTERSUMP CATWALK DOORS



WATER SURFACE EL 418



WATER SURFACE EL 422

NOTE: VELOCITIES GIVEN IN FEET PER SECOND.

APPROACH VELOCITIES
MEASURED 1 FT ABOVE SUMP FLOOR
ALL PUMPS OPERATING AT 134 CFS EACH
RECOMMENDED DESIGN SUMP

END

1-87

DTIC